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DESIGN

OF A

REINFORCED CONCRETE ARCH BRIDGE

BY

JOHN HENNING ANDERSON

THESIS

FOR THE

DEGREE OF BACHELOR OF SCIENCE

IN

CIVIL ENGINEERING

COLLEGE OF ENGINEERING

UNIVERSITY OF ILLINOIS

1914

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UNIVERSITY OF ILLINOIS

COLLEGE OF ENGINEERING

May 25, 1914.

I hereby recommend that the thesis prepared under my direction by JOHN HENNING ANDERSON entitled DESIGN OF A REINFORCED CONCRETE ARCH BRIDGE be accepted as fulfilling this part of the requirements for the degree of Bachelor of Science in Civil Engineering.

W. M. Wilson

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Recommendation concurred in:

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DESIGN OF A REINFORCED CONCRETE ARCH BRIDGE.

1. ACKNOWLEDGMENT.

This thesis was prepared by the author in conjunction with Mr. Frank Thomas Sheets, and represents the joint effort of both. With the exception of this acknowledgment, both theses are exact duplicates, and were prepared, as such, in accordance with the regulations of the University.

2. GENERAL STATEMENTS.

This bridge is designed to span the Rock River between Rock Island County and Henry County near the town of Colona, Illinois. At present no bridge exists at this point. A ferry is used for the transportation of traffic.

The field survey of the bridge site, used in connection with this design, was made by the engineers of the State Highway Commission of Illinois, and the information was taken from a general plan prepared by the Commission. Acknowledgment is due to the State Highway Commission for the complete information which was so gladly furnished.

3. CONDITIONS GOVERNING THE DESIGN.

The river bed is underlaid by a stratum of hard limestone located very close to the surface. In one place there is a pocket in which the rock has not been located. The river bed itself consists of a mixture of sand and gravel which covers the rock for depths varying from two to nine feet. On either shore this material is overlaid by a deposit of loam. The river at this point is fairly swift, and is not navigable. Plenty of material is available at the bridge site for building approach grades and for other fills. The banks on either shore are low and flat. The bridge is designed for country traffic.

4. SPECIFICATIONS.

The specifications of the State Highway Commission, "Edition of 1913", are followed in the design.

The loadings used are as follows:

Live load, 125 lb. per sq. ft. of roadway.

Earth fill, 100 lb. per cu. ft.

Macadam, 100 lb. per cu. ft.

Concrete, 150 lb. per cu. ft.

The allowable unit stresses used are as follows:

Steel in tension, 16,000 lb. per sq. in.

Concrete in compression, 800 lb. per sq. in.

Concrete in shear, 40 " " " "

Bond stress, 80 " " " "

The proportions of concrete to be used are, as stated in the specifications;

Class A. 1 part cement, 2 1/2 parts sand, 4 parts crushed stone.

Class B. 1 " " 3 " " 5 " " "

Class A concrete is to be used in the arch rib, spandrel walls, abutment wing walls and all other places where reinforced concrete is used, with the exception of the anchor piers and the bottom of the abutments. Class B concrete is to be used in the piers and abutments only.

5. TYPE OF BRIDGE.

Since the river bed is underlaid by a stratum of hard limestone, no difficulty will be encountered in securing good foundations. In view of the fact that the rock is so near the surface, the concrete arch presents itself as the most logical type of bridge to use. This solid foundation makes an ideal situation for such a structure. It is believed that the arch will compete in cost with other forms of construction, and when the aesthetic side of the problem is considered, the arch bridge stands out above all others as an economical and beautiful structure.

6. SPAN OF ARCH.

The choice of the span length involves a number of important considerations. A reference to Plate 16 shows the location of the pocket in which the rock was not located. It is necessary that such a length be used as will span this gap and afford firm foundations for the piers and abutments on either side. This will require a span of at least eighty feet.

In order to keep the waterway as clear as possible, it is desirable that the number of piers be reduced to a minimum. Furthermore, reducing the number of piers greatly reduces the foundation cost. On the other hand, increasing the length of the span increases the cost of the superstructure. This is especially true in this case because the rise of the arch is limited. The fact that the rock foundation is so near the surface makes the cost of the substructure relatively small. For these reasons, the most economical length of span is comparatively short. After a careful study of existing structures and after making some estimates of the relative costs for different lengths, the eighty-foot span is chosen.

The same span length is used thruout the entire bridge in order that, in the construction the same centering may be used repeatedly. It is evident that the cost may thus be materially reduced without in any way sacrificing the beauty or strength of the bridge.

7. WIDTH OF THE ROADWAY.

It is obvious that an excessive width of roadway will greatly increase the cost of the bridge. For this reason, the roadway should be made as narrow as possible. The length of the bridge necessitates the accommodation of two lines of traffic. The minimum width which will do this satisfactorily is eighteen feet, and this width is therefore adopted.

8. SHAPE OF THE ARCH.

A number of conditions lead to the choice of an elliptical arch having a small ratio of rise to span. The banks on either shore are low and flat. It is undesirable to elevate the roadway more than necessary because of the heavy approach fills which would inevitably result. The roadway must then be kept as low as possible. It is however necessary to accommodate the periods of highwater, and for safety it is not desirable to have the water rise to any great extent above the springing line of the arch. This results in the choice of an arch having a small ratio of rise to span. Plate 16 shows the location of highwater with reference to the springing line.

To give maximum waterway under these conditions, it is necessary that an elliptical form of arch be used. For ease of construction, an arch having a three-centered intrados and extrados is adopted, which very closely approximates the truly elliptical form.

9. TYPE OF SPANDREL.

The elements which govern the type of spandrel to be adopted are economy and appearance.

A careful analysis of the cost of the filled spandrel and the open spandrel types shows that the former is by far the cheaper type of construction. The necessity for floor slabs and other expensive construction runs the cost of the open type greatly in excess of the filled form for such a low height of arch. The abundance of material at the bridge site minimizes the cost of filling the spandrel. The filled spandrel is much more pleasing in appearance. The low height necessitates a solid side wall for the sake of appearance so that, if the open type of spandrel were used, it would have to be disguised.

10. ANCHOR PIERS.

Three of the piers are designed so that if the span on one side of the pier were removed the pier would take the unbalanced dead load thrust of the other span. The bridge is thus divided up into four parts each of which will stand as a separate unit.

One of the reasons for designing the bridge in this manner is to obtain ease and economy in construction. With this design, the centering may be used for one unit and then taken down and moved ahead for the other units.

Another reason for the use of this design is to guard against the emergency of one arch failing. In that event, only the arches remaining in that unit would fail and the remainder of the bridge would be left intact. The location of these anchor piers is shown on Plate 16.

11. CAMBER IN THE BRIDGE.

An examination of Plate 16 will reveal the fact that the top of the hand rail, the crown of roadway and the tops of the piers at the center of the bridge are nineteen inches above the elevation of these points at the abutments.

One reason for using this camber for the bridge is to provide ample drainage of the roadway. The method of doing this is shown on Plate 19.

Another important factor which leads to the use of this camber is that of appearance. If the bridge were built perfectly level on top the hand rail would seem to sag at the center. The camber relieves this effect and gives the bridge a very pleasing appearance.

All of the arches are identical, and the desired camber is obtained by placing the springing lines of the various arches on the arc of a parabola. The calculations for determining the ordinates of this parabola are shown in Plate I.

12. DESIGN OF ARCH.

The elastic theory of arches as presented by Professor McKibben in Taylor and Thompson's, "Concrete, Plain and Reinforced", and by Turneaure and Maurer's, "Principles of Reinforced Concrete Construction" is used in this design.

The method presented by Professor McKibben requires that the arch ring be subdivided into a number of parts, usually twenty as in this case, such that the ratio of their length to the average value of the moment of inertia therein shall be constant. Plate 2 shows the computations and Plate 17 shows the graphical work by which this is accomplished. For this method see Turneaure and Maurer's, "Principles of Reinforced Concrete Construction", page 354.

Plate 3 shows the computations for the loads used in the design.

Table I on Plate 4 shows the computations for the arch when the live-load covers the entire span. Table II on Plate 5 shows the computations for live load over the right half of the arch only, while Table III on Plate 6 shows the computations when dead load alone is acting on the structure.

In Table IV on Plate 7 the combinations of shears and moments due to live and dead loads, temperature, and rib shortening are given. These values are used in the final investigation of the arch.

Plates 8 and 9 contain the computations relating to the investigation of typical sections of the arch. These investigations are for the purpose of determining the values of the stresses in the concrete and steel.

13. DESIGN OF ABUTMENTS.

Plates 10 and 11 show the computations for abutment #1 while Plate 12 shows the computations for abutment #2. Plate 18 shows the graphical work relating to the design of the abutments. As drawn on this plate the resultant strikes at the outer edge of the middle third of the base, thus causing the pressure at the outer edge to be twice the average value. Since the foundation is solid rock, the pressures attained are safe. The outer part of the abutment acts as a cantilever, and since tensile stresses are developed, steel is introduced. Shearing stresses govern the depth of this cantilever. The details of the abutments are shown on Plates 20 and 21.

14. DESIGN OF ANCHOR PIERS.

On Plates 13 and 14 are given the computations relating to the design of the anchor piers, while the graphical work connected therewith is shown on Plate 18. The resultant of the forces falls entirely outside of the base. It is impracticable to construct a gravity structure which will be capable of resisting the overturning moment due to the unbalanced dead load thrust. The base width to be used should be as small as possible for the sake of economy, while at the same time it should be sufficiently great to reduce the stress in the anchor bolts to a practicable value. A base width of twenty feet is used. As shown on Plate 23, anchorage is secured by means of steel bars set in holes drilled in the rock and extending up into the base. These holes are to be filled with portland cement after the bars have been inserted. The bond stress developed by the bars is the feature that controls the depth of the holes. The parts of the base extending beyond the pier proper are reinforced to act as cantilevers, while the body of the pier is reinforced to act as a cantilever on top of the base.

15. ARCH SPANDREL WALL.

Plate 15 shows the computations relating to the arch spandrel walls. As shown on Plate 19, this wall is designed to act as a cantilever resisting an equivalent fluid pressure of 14.7 lbs. per cu. ft. See Turneaure and Maurer, page 373.

16. EXPLANATION OF DRAWINGS.

Plate 16 shows the general plan and elevation of the entire bridge, and gives complete information in regard to high and low water, profile and character of river bed, etc., as furnished by the engineers of the Illinois State Highway Commission. This plate also shows the elevations of the tops and bottoms of piers, crown of roadway, etc.

Plate 19 shows complete details of the arch ring. The longitudinal reinforcing bars used amount to one per cent of the area at the crown. The transverse bars are used to take up temperature stresses, and to tie the longitudinal bars together. The arch ring is made two inches thicker at the center than at the sides to provide drainage. The longitudinal steel is placed a distance from the face of the concrete equal to one-tenth the depth of the arch ring.

On Plate 22 are shown the details of the plain concrete piers. The upstream edge is protected by a 6" x 6" x 3/4" angle which serves as an ice cutter. The thickness of the piers was kept as small as possible for the sake of economy, while at the same time it was made to harmonize with the general appearance of the arches.

17. CONCLUSIONS.

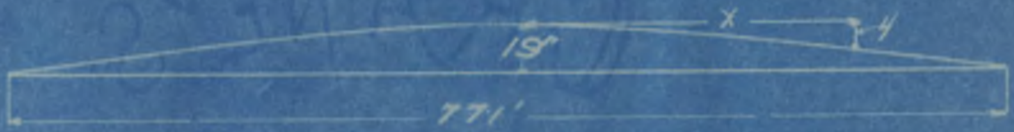
In reviewing the design as a whole, the writers feel that all the governing conditions have been met, and that a structure has been obtained, in which the aesthetic principles of designing have been carefully weighed in their economic relation to cheapness of construction. It is possible that the structure might be cheapened by using an arch of greater rise, and allowing the high water to rise to a higher point above the springing line. For the sake of additional safety this was not done. This point is largely a matter of opinion, and can only be definitely decided by the elements themselves.

Thruout the entire design safe and conservative stresses have been obtained and yet no material has been needlessly wasted. The arch itself is decidedly on the side of safety. All the elements of the design have been so proportioned that they have the same degree of strength. An effort has been made to keep the bridge well balanced without having one part excessively strong and another dangerously weak.

In conclusion it may be said that this design attempts to follow the best modern practice relating to concrete bridges and involves no eccentric ideas or peculiar features which have not been tried out by actual construction and satisfactory service.

VERTICAL CURVE

Tops of piers to be placed on a parabola with rise of 19" at the center.



$$\frac{X^2}{\left(\frac{771}{2}\right)^2} = \frac{y}{1.6}, \quad y = 0.0000108\,X^2$$

86'-0" and 86'-6" ℄ to ℄ of piers

X	Y
43.25'	0.018'
129.25	0.168
215.75	0.465
302.25	0.910
385.50	1.600

Ele of Crown of Roadway

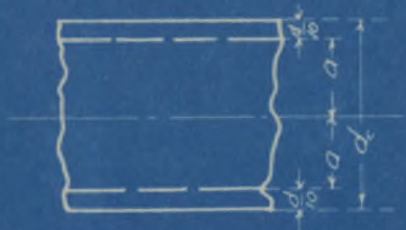
Abutment #1	54.00
Pier #1	54.60
" #2	55.15
" #3	55.45
" #4	55.60
" #5	55.60
" #6	55.45
" #7	55.15
" #8	54.60
Abutment #2	54.00

Ele at Springing Line

Abutment #1	41.50
Pier #1	42.10
" #2	42.65
" #3	42.95
" #4	43.10
" #5	43.10
" #6	42.95
" #7	42.65
" #8	42.10
Abutment #2	41.50

PLATE # 2

DIVISIONS OF ARCH RING



d_c = depth at crown in ft.
$n = 15 \quad p = .01$
b_c = width at crown ft.
I = moment of inertia of entire section about axis.
I_c = moment of inertia of concrete
I_s = moment of inertia of steel
b = width at any section
d = depth " " "
n = moduli ratio $= \frac{E_s}{E_c}$
p = percent steel at crown.

$$I = I_c + I_s$$
$$= \frac{bd^3}{12} + np\,b_c d_c a^2$$

$b_c = 1$ $\quad a = .4d$
$b = 1$

$$I = \frac{d^3}{12} + 15 \times .01 \times 1 \times 1.5 \times .16\,d^2$$
$$= \frac{d^3}{12} + .036\,d^2 = .083\,d^3 + .036\,d^2$$

No. of Div.	Depth	d^3	I_c	d^2	I_s	I	$i = \frac{1}{I}$
10	1.5	3.38	.280	2.25	.081	.361	2.77
9	1.55	3.72	.308	2.40	.086	.394	2.54
8	1.60	4.10	.340	2.56	.092	.432	2.32
7	1.68	4.75	.394	2.82	.101	.495	2.02
6	1.82	6.02	.50	3.31	.119	.619	1.62
5	1.96	7.51	.624	3.84	.138	.762	1.31
4	2.15	9.95	.825	4.61	.166	.991	1.01
3	2.41	14.00	1.16	5.81	.209	1.369	.73
2	3.11	30.20	2.51	9.69	.384	2.858	.35
1	4.15	71.50	5.94	17.20	.620	6.56	.15

$\frac{ds}{I} = \frac{s i_a}{n}$ See Turneaure & Maurer.

ds = length of div.
I = moment of inertia
s = length of ½ arch axis
i_a = average of i

$s = 43 \quad i_a = 1.48 \quad n = 10$

$\frac{ds}{I} = \frac{43 \times 1.48}{10} = 6.36$ = Constant $\quad ds.i = 6.36$

For divisions see diagram on graphical sheet for arch.

$A_s = .01 \times 18 \times 12 = 2.16$ sq. in $\quad$ 1.08 in each layer.

Use ¾ in. sq. twisted bars spaced 6¼ in. cts. = 1.08

COMPUTATIONS OF LOADINGS

Earth = 100 lb/cu ft Macadam = 100 lb/cu ft Concrete = 150 lb/cu ft

Uniform Live Load = 125 lb per sq ft

Section	Dead Load Lbs.	Live Load	Total
1a	Concrete $\frac{3.8\times 4.6}{2} = 8.74$ $\frac{5.5\times 3.7}{2} = \underline{10.20}$ $18.94 \times 150 = 2840$ Earth $\frac{5.7+8.2}{2}\times 4.9 \times 100 = \underline{3410}$ Total 6250	612	6862
1b.	Concrete $\frac{5.2\times 3.4}{2} = 8.84$ $\frac{4.5\times 2.5}{2} = \underline{5.63}$ $14.47 \times 150 = 2170$ Earth $\frac{4.2+5.7}{2}\times 4.3 \times 100 = \underline{2130}$ Total 4300	537	4837
1c	Concrete $\frac{4.8\times 2.5}{2} = 6.00$ $\frac{4.7\times 2.2}{2} = \underline{5.17}$ $11.17 \times 150 = 1675$ Earth $\frac{2.3+4.2}{2}\times 4.6 \times 100 = \underline{1495}$ Total 3170	575	3745
2	Concrete $\frac{5.5\times 2.15}{2} = 5.92$ $\frac{5.5\times 1.95}{2} = \underline{5.36}$ $11.28 \times 150 = 1690$ Earth $\frac{2.5+3.4}{2}\times 5.4 \times 100 = \underline{1591}$ Total 3281	675	3956
3	Concrete $\frac{4.25\times 1.95}{2} = 4.15$ $\frac{4.2\times 1.8}{2} = \underline{3.78}$ $7.93 \times 150 = 1190$ $\frac{2+2.5}{2}\times 4.2 \times 100 = \underline{945}$ Total 2135	525	2660
4	Concrete $\frac{1.8+1.7}{2}\times 3.5 \times 150 = 917$ Earth $\frac{1.7+2.0}{2}\times 3.4 \times 100 = \underline{628}$ Total 1545	425	1970
5.	C $\frac{1.65+1.7}{2}\times 3.0 \times 150 = 753$ E $\frac{1.4+1.7}{2}\times 3.0 \times 100 = \underline{465}$ Total 1218	375	1593
6	C $\frac{1.6+1.65}{2}\times 2.8 \times 150 = 682$ E $\frac{1.3+1.4}{2}\times 2.8 \times 100 = \underline{378}$ T = 1060	350	1416
7	C $\frac{1.55+1.60}{2}\times 2.6 \times 150 = 614$ $\frac{1.2+1.3}{2}\times 2.6 \times 100 = \underline{325}$ 939	325	1264
8	C $\frac{1.5+1.55}{2}\times 2.5 \times 150 = 572$ E $1.1\times 2.5 \times 100 = \underline{275}$ 847	315	1162
9	C $1.5\times 2.35\times 150 = 529$ E $2.3\times 1.0\times 100 = \underline{230}$ 759	288	1047
10	C $1.5\times 2.3\times 150 = 516$ E $1.0\times 2.3\times 100 = \underline{230}$ 746	288	1034

PLATE # 4

TABLE # I

LIVE LOAD over WHOLE SPAN

Point	X	Y	X^2	Y^2	M_L	M_R	$M_L X$	$M_R X$	$M_L Y$	$M_R Y$
10-11	1.15	0.00	1.32	0.00	×		×		×	
9-12	3.52	0.00	12.4	0.00	2,450		8,630		×	
8-13	5.95	0.11	35.4	0.01	7,510		44,700		827	
7-14	8.50	0.25	72.1	0.06	15,780	Same as M_L	134,000	Same as $M_L X$	3,945	Same as $M_L Y$
6-15	11.2	0.40	125.5	0.16	27,970		314,000		11,188	
5-16	14.1	0.65	198.8	0.42	45,100		636,000		29,300	
4-17	17.4	0.97	302.8	0.94	69,900		1,217,000		67,500	
3-18	21.2	1.45	449.6	2.10	105,950		2,742,000		153,700	
2-19	26.0	2.20	676.0	4.84	164,250		4,270,000		361,900	
1-20	35.4	4.70	1255.0	22.10	331,600		11,720,000		1,558,000	
Σ	144.42	10.73	3128.9	30.63	770,570		20,586,330		2,186,360	

FORMULAE, LL + DL

$$H_c = \frac{m\Sigma M_R Y + m\Sigma M_L Y - \Sigma M_R \Sigma Y - \Sigma M_L \Sigma Y}{2\times\left[m\Sigma Y^2 - (\Sigma Y)^2\right]}$$

$$V_c = \frac{\Sigma M_L X - \Sigma M_R X}{2\Sigma X^2}$$

$$M_c = \frac{\Sigma M_R + \Sigma M_L - 2H_c\Sigma Y}{2m}$$

$$H_c = \frac{10\times 4,372,720 - 10.73\times 1,541,140}{2\times(10\times 30.63 - 115.13)} = 71,100 \text{ lbs}$$

V_c = zero

$$M_c = \frac{1,541,140 - 2\times 71,100\times 10.73}{20} = +767 \text{ lb ft.}$$

M_L at 20-c = 246,300 lb.ft.
M_L at 20-a = 435,300 lb.ft.
M_L at springing = 485,800 lb.ft.

TABLE NO. II

DEAD LOAD AND LIVE LOAD OVER RIGHT HALF.

Point	x Ft.	y Ft.	x^2	y^2	M_L	M_R	$M_L x$	$M_R x$	$M_L y$	$M_R y$
10-11	1.15	0	1.32	0	0	0	0	0	0	0
9-12	3.52	0	12.40	0	1770	2450	6230	8630	0	0
8-13	5.95	.11	35.40	.01	5430	7510	32300	44700	596	827
7-14	8.50	.25	72.10	.06	11430	15780	97300	134000	2857	3945
6-15	11.20	.40	125.50	.16	20320	27970	227800	313400	8128	11188
5-16	14.10	.65	198.8	.42	32920	45100	464500	636000	21400	29300
4-17	17.40	.96	302.8	.92	51320	69,900	895000	1217000	49400	67500
3-18	21.20	1.45	450.0	2.10	78920	105950	1660000	2242000	113500	153,500
2-19	26.00	2.20	676.0	4.84	122720	164250	3190000	4270,000	270000	361,900
1-20	35.40	4.68	1255.0	22.00	254220	331700	9000000	11720,000	1194000	1558000
Totals	144.42	10.70	3129.32	30.51	578,450	770610	15573130	20585730	1659881	2186160

M_L 1c = 186720
" 1b = 254220
1a = 338220
Sp. Line = 380220

$$H_c = \frac{10 \times 2,186160 + 10 \times 1659881 - 770610 \times 10.7 - 578450 \times 10.7}{2\,[\,10 \times 30.51 - 10.7 \times 10.7} = 63,100.$$

$$V_c = \frac{15573130 - 20585730}{2 \times 3129.32} = -800.$$

$$M_c = \frac{770,610 + 578450 - 2 \times 63100 \times 10.7}{20} = -50$$

e = neglected

TABLE NO III
DEAD LOAD ONLY

Point	x ft.	y ft.	x^2	y^2	M_L	M_R	$M_L x$	$M_R x$	$M_L y$	$M_R y$
10-11	1.15	0	1.32	0	0	Same	0	Same	0	same
9-12	3.52	0	12.4	0	1770	as	6240	as	0	as
8-13	5.95	0.11	35.4	.01	5430	M_L	32300	$M_L x$	596	$M_L y$
7-14	8.50	.25	72.1	.06	11420		97200		2860	
6-15	11.20	.40	125.5	.16	20310		227500		8130	
5-16	14.10	.65	198.8	.42	32930		464500		21400	
4-17	17.40	.96	302.8	92	51330		894000		49300	
3-18	21.20	1.45	450.0	2.10	78330		1660000		113500	
2-19	26.00	2.20	676.0	4.84	122730		3190000		270000	
1-20	35.40	4.68	1255.0	22.00	254230		9000000		1190000	
Totals	144.42	10.70	3129.32	30.51	578480		15571740		1655786	

M_L at 20 c = 186730

20 b = 254230

20 a = 338230

Sp. Line = 380230

$$H_c = \frac{10 \times 3,311572 - 2 \times 578480 \times 10.7}{382} = 54300$$

Eccentricity at springing line.

Thrust 57700 Arm 8.40

$M = H_c y - M_L$

$= 54300 \times 8.4 - 380230 = 75070$

$$e = \frac{75070}{57700} = 1.30$$

TABLE # IV

FINAL MOMENTS and THRUSTS

	LIVE + DEAD LOADS				TEMPERATURE		RIB SHORTENING	
Point	Mom	Shear	Thrust	Ecc	Mom	Thrust	Mom	Thrust
Left Springing	+123,000	23,400	64,500	+1.91'	+77,350	+7,600	-60,900	-6,000
1-b	+13,500	11,400	64,100	+0.21'	+36,750	+9,500	-29,000	-7,500
20-c	-25,000	1,150	64,800	-0.38'	+22,650	+10,100	-17,800	-7,950
Right Springing	+79,450	21,300	66,700	+1.19'	+77,350	+7,600	-60,900	-6,000
Crown	+23	-770	62,750	0.00'	-11,150	+10,400	+8,800	-8,200
4	-3,900		63,300	-0.06'	-1,050	+10,300	+760	-8,100
18	+1,350		63,600	+0.02'	+3,950	+10,300	-3,100	-8,100
Crown	+767	Zero	71,100	+0.011	-11,150	+10,400	+8,800	-8,200

ᒻFor L.L. over whole arch

$M = M_c + V_c X + H_c y - M_L$ (Left ½ of arch)

$M = M_c - V_c X + H_c y - M_R$ (Right ½ of arch)

(L.S.) $M = -770 \times 41.4 + 62{,}750 \times 8.5 - 378{,}500 = +123{,}000$ lb.ft.

(1-b) $M = -770 \times 35.4 + 62{,}750 \times 4.7 - 254{,}250 = +13{,}500$ lb.ft.

(20-c) $M = +770 \times 30.8 + 62{,}750 \times 3.15 - 246{,}300 = -25{,}000$ lb.ft.

(R.S.) $M = +770 \times 41.4 + 62{,}750 \times 8.5 - 485{,}800 = +79{,}455$ lb.ft.

(4) $M = -770 \times 17.4 + 62{,}750 \times 0.97 - 51{,}400 = -3{,}900$ lb.ft.

(18) $M = +770 \times 21.2 + 62{,}750 \times 1.45 - 105{,}950 = +1{,}350$ lb.ft.

Temp. $\begin{cases} H_c = \frac{I}{3} \cdot \frac{\epsilon t L m E}{2[m\Sigma y^2 - (\Sigma y)^2]} = \frac{1}{636} \cdot \frac{0.0000055 \times 20 \times 80 \times 10 \times 2{,}000{,}000 \times 144}{382.4} = 10{,}400\# \\ M_c = -\frac{H_c \Sigma y}{m} = -\frac{10400 \times 10.73}{10} = -11{,}150 \text{ lb.ft.} \end{cases}$

$M = M_c + H_c y$ = mom. due to temperature

(L.S.) $M = -11{,}150 + 10{,}400 \times 8.5 = +77{,}350$ lb.ft.

(1-b) $M = -11{,}150 + 10{,}400 \times 4.60 = +36{,}750$ lb.ft.

(4) $M = -11{,}150 + 10{,}400 \times 0.97 = -1{,}050$ lb.ft.

Average compression thruout arch due to dead and live load normal thrust = 25,000 lbs per sq.ft. = f

Rib Shortening $\begin{cases} H_c = -\frac{I}{3} \cdot \frac{f L m}{2[m\Sigma y^2 - (\Sigma y)^2]} = -\frac{1}{636} \cdot \frac{25{,}000 \times 80 \times 10}{382.4} = -8{,}200\# \\ M_c = -\frac{H_c \Sigma y}{m} = \frac{+8200 \times 10.73}{10} = +8{,}800 \text{ lb.ft.} \end{cases}$

(1-b) $M = +8{,}800 - 8200 \times 4.6 = -29{,}000$ lb.ft.

$Ecc = \frac{Mom}{Thrust}$

INVESTIGATION of SECTIONS

Left Springing

LL + DL + RS

Mom	Thrust	
+123,000	+64,500	LL + DL
− 60,900	− 6,000	RS
+62,100	+58,500	

$e = \frac{M}{N} = 1.06'$ $p = 0.0033$

$\frac{e_0}{h} = 0.174$ therefore part of section is in tension

$\frac{e}{h} = 0.236$, $K = 0.81$ from Fig 179

$Ca = 0.103$ from Fig 180

$fc = \frac{M}{Cabh^2} = \frac{62,100}{0.103 \times 1 \times 20.25} = 29,800\#^{\square'}$

$= 207\#^{\square''}$ comp in concrete

$fs = nfc\frac{d-Kh}{Kh} = 15 \times 207 \frac{4.05-3.64}{3.64}$

$= 347\#^{\square''}$ tension in steel

LL + DL + RS + Temp

Mom	Thrust	
+62,100	+58,500	
+77,350	+ 7,600	+ Temp 20°
+139,450	+66,100	

$e = \frac{M}{N} = 2.11'$, $p = 0.0033$

$\frac{e}{h} = 0.47$, $K = 0.44$

$Ca = 0.096$, $fc = 488\#^{\square''}$

$fs = 7,660\#^{\square''}$

CROWN

Mom	Thrust	
+23	+62,750	LL + DL
+8800	− 8200	RS
+8823	+54,550	

$e = \frac{M}{N} = 0.162$, $p = 0.01$

$\frac{e}{h} = 0.108$

$\frac{e_0}{h} = 0.1865$ (Compression over entire area)

$Ce = 1.47$ from fig 177

$fc = \frac{NCe}{bh} = \frac{54,550 \times 1.47}{1 \times 1.5} = 53,500^{\square'}$

$= 371\#^{\square''}$ comp in concrete

Mom	Thrust	
+8823	+54,550	
−11,150	+10,400	+ Temp 20°
−2,327	+64,950	

$e = \frac{M}{N} = -0.036$, $p = 0.01$

$\frac{e}{h} = 0.024$

$Ce = 1.01$

$fc = 304\#^{\square''}$

Figs. referred to above are found in Taylor and Thompson's "Concrete, Plain and Reinforced"

Above investigation is for LL over right half of arch only. For LL over whole arch and a fall in temp of 20° the following values are obtained:

Mom. = 20,717, $e = 0.394$, $K =$ ~~[illegible]~~ .81 $fc =$ ~~[illegible]~~ $519\#/\square''$

Thrust = 52,500, $\frac{e}{h} = 0.263$ $fs =$ ~~[illegible]~~ $1870\#/\square''$

INVESTIGATION of SECTIONS

Section 1-b

LL + DL + R.S.

Mom.	Thrust	
+13,500	+64,100	LL + DL
−29,000	− 7500	R.S.
−15500	+56,600	

$e = \frac{-15500}{56,600} = -0.27$ $p = 0.00051$

$\frac{e}{h} = \frac{0.27}{2.95} = 0.093$

$\frac{e_0}{h} = 0.1775$ (No tension)

$C_e = 1.11$

$f_c = \frac{N C_e}{bh} = \frac{56,600 \times 1.11}{1 \times 2.95} =$

20,900#°′ = 146#°″

LL + DL + R.S. + Temp

Mom.	Thrust	
−15,500	+56,600	
−36,750	− 9,500	−Temp.
−52,250	+47,100	

$e = \frac{M}{N} = 1.11'$, $\frac{e}{h} = \frac{1.11}{2.95} = 0.376$

$p = 0.0051$ $\frac{e_0}{h} = 0.1775$

(Tension over part of section)

$K = 0.565$, $C_a = 0.109$

$f_c = \frac{M}{C_a b h^2} = 54,100$#°′ = 376#°″

$f_s = n f_c \frac{d - Kh}{Kh} = 3,370$#°″

Section 4

Mom.	Thrust	LL + DL
−3,900	+63,300	LL + DL
+ 760	− 8,100	R.S.
−3140	+55,200	

$e = \frac{3140}{55,200} = 0.057$, $p = 0.0085$

$\frac{e}{h} = \frac{0.057}{1.75} = 0.033$

$\frac{e_0}{h} = 0.184$ (No tension)

$C_e = 1.56$

$f_c = \frac{55200 \times 1.56}{1 \times 1.75} = 46,400$#°′ =

322#°″

Mom.	Thrust
−3,140	+55,200
−1,050	+10,300
−4,190	+65,500

$e = \frac{M}{N} = 0.062$, $p = 0.0085$

$\frac{e}{h} = 0.035$, $\frac{e_0}{h} = 0.184$

(No tension)

$f_c = 406$#°″

CROWN

Fall in Temperature

Mom.	Thrust
+8,823	+54,550
+11,150	−10,400
+19,973	+44,150

$e = \frac{19973}{44,150} = 0.45$, $\frac{e}{h} = \frac{0.45}{1.5} = 0.3$

$p = 0.01$, $\frac{e_0}{h} = 0.1865$ (Tension over part of crown)

$K = 0.72$ $C_a = 0.127$

$f_c = \frac{19973}{0.127 \times 2.25} = 69,900$#°′ = 486#°″

$f_s = 15 \times 486 \frac{1.35 - 1.08}{1.08} = 1850$#°″

PLATE #10

COMPUTATIONS FOR ABUTMENT NO. 1

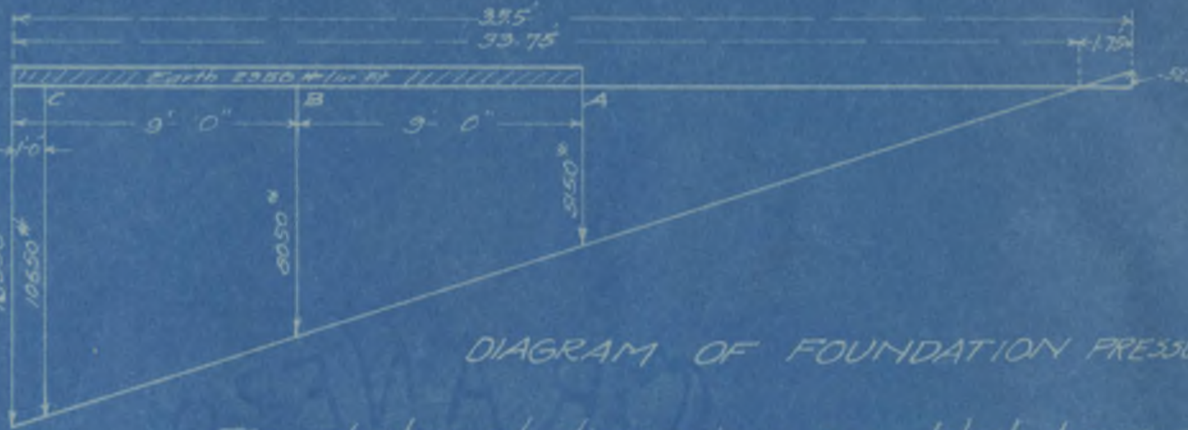


DIAGRAM OF FOUNDATION PRESSURE

För abutment dimensions see detail drawings.
För graphical work see graphic sheet.

$P = \frac{2}{3}\frac{V}{\left(\frac{l}{2} - d\right)}$ $\quad d = 6.50'$ $\quad V = 185{,}500^{\#}$

$P_1 = \frac{2}{3}\left(\frac{185500}{\frac{33.5}{2} - 6.5}\right) = \text{———} = 10950\ ^{\#}/\square'$ at toe.

$P_2 = -5230 \times 0.1 = -523\ ^{\#}/\square'$ at heel.

$\frac{10950 + 5150}{2} \times 18 = 144900^{\#}$ = total shear at A

$\frac{144900}{23 \times 144} = 44\ ^{\#}/\square''$ = unit shear at A.

$\frac{10950 + 8050}{2} \times 9 = 85500$ = total shear at B

$\frac{85500}{13.3 \times 144} = 44.7\ ^{\#}/\square''$ unit shear at B.

$\frac{10950 + 10650}{2} \times 1 = 10800$ total shear at C.

$\frac{10800}{4.9 \times 144} = 15\ ^{\#}/\square''$ unit shear at C.

Moment at A.

$5150 \times 18 \times 9 + \frac{5800 \times 18}{2} \times 12 - 2350 \times 18 \times 9 = 1{,}080{,}000$ ft lbs.
$= 12{,}960{,}000$ in. lbs.

Effective depth = 22.0 = d

$R = \frac{M}{bd^2} = \frac{12960000}{12 \times 264 \times 264} = 16.5$ $\quad f_s = 16000\ ^{\#}/\square''$ $\quad p = .09\%$
$f_c = 200\ ^{\#}/\square''$

$A_s = 12 \times 264 \times .0009 = 2.85\ \square''$ per ft = amount of steel.
Use $1\frac{1}{4}''\square$ bars $6\frac{1}{2}''$ cts. = $2.90\ \square''$

Moment at B. $8050 \times 9 \times 4.5 + \frac{3900 \times 9}{2} \times 6 - 2350 \times 9 \times 4.5 = 309150$ ft. lb.
$= 3709800$ in. lb.

$d = 12.5'$ $\quad R = \frac{3709800}{12 \times 150 \times 150} = 13.7$ $\quad f_s = 16000$ $\quad f_c = 250$ $\quad p = .00085$

$A_s = 12 \times 150 \times .00085 = 1.53\ \square''$
Use $1\frac{1}{4}''$ sq. bars 13" cts. = $1.45\ \square''$

COMPUTATIONS FOR ABUTMENT NO.1. continued

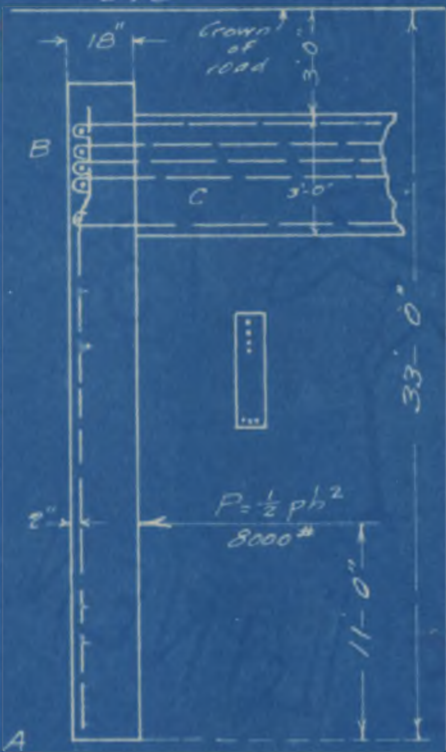
Earth pressure used in graphical analysis.
Formulae from Taylor & Thompson pp. 666

$P = w(H^2 - h^2)C_p$ $\qquad x = \dfrac{(H-h)^2}{6(H+h)}$

$C_p = 0.19 \quad H = 37' \quad h = 8'$

$P = 100(\overline{37}^2 - \overline{8}^2) \times .19 = 100 \times 1906 \times 0.19 = 24800^{\#}$

$X = \dfrac{840}{270} = 3.1$



DESIGN OF WING WALLS

Use equivalent fluid pressure of 14.7.
Consider A-B as vertical slab supported at top by horizontal beam B-D running between struts S-S. and supported at bottom by ground or the abutment.

$P = \frac{1}{2} Ph^2 = \frac{1}{2} \times 14.7 \times \overline{33}^2 = 8000^{\#}$ $\qquad n = 15$

$M = .064\, ph^3 = .064 \times 14.7 \times \overline{33}^3 \times 12 = 405000$ in.lb.

$R = \dfrac{405000}{12 \times 16 \times 16} = 132$ $\quad f_s = 16000 \quad f_c = 750 \quad pj = .00826 \quad p = .0096$

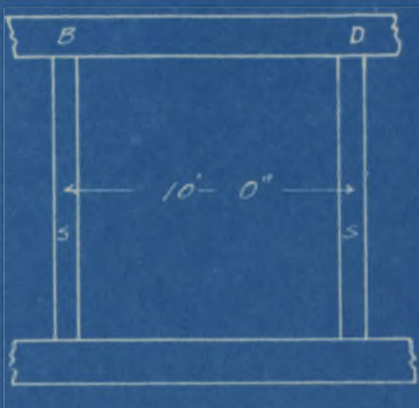
$A_s = .0096 \times 12 \times 16.0 = 1.84 \square''$
Use 1"□ bars 6½" cts = 1.85 □"

Design beam BD for span of distance between struts and a uniform load equal to reaction of vert beam A-B

$w = 8000 \times \dfrac{11}{28.5} = 3090$

$M = \frac{1}{8} \times 3090 \times 10 \times 10 \times 12 = 464000$ in.lb.

$R = \dfrac{464000}{18 \times 16 \times 16} = 100.05$ $\quad pj = .0063 \quad p = .0072 \quad f_s = 16000 \quad f_c = 620$



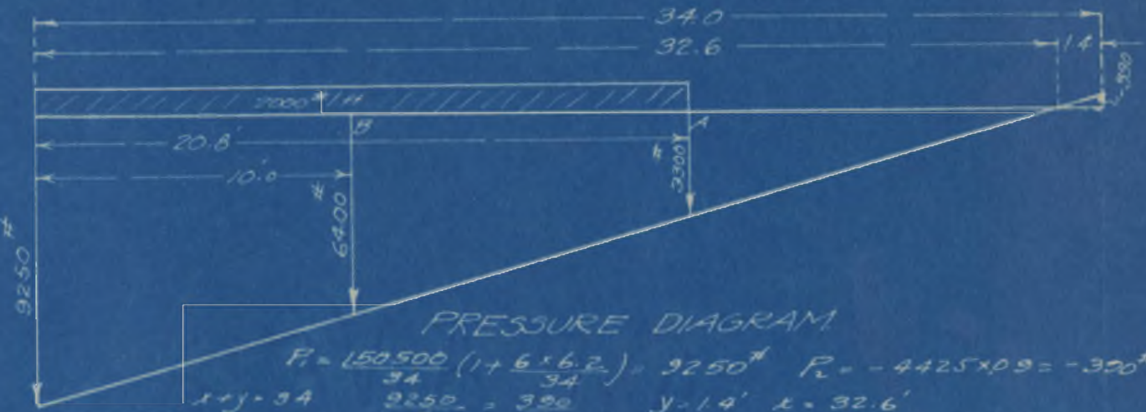
$A_s = .0072 \times 18 \times 16 = 2.08$ sq.in.
Use 4 - 1"□ rods.

Design steel in strut S to take reactions of beam BD. $f_s = 16000$

$\dfrac{30900}{16000} = 1.93 \square''$ reqd Use 4 - 1"□ rods.

Also place 3 - 1"□ rods in bottom of strut to prevent cracking.
For temperature steel in wall use ½"□ bars 12" cts. horizontally.

COMPUTATIONS FOR ABUTMENT NO. 2.



PRESSURE DIAGRAM.

$P_1 = \frac{158500}{34}\left(1 + \frac{6 \times 6.2}{34}\right) = 9250^{\#}$ $P_2 = -4425 \times 0.9 = -350^{\#}$

$x + y = 34$ $\frac{9250}{x} = \frac{350}{y}$ $y = 1.4'$ $x = 32.6'$

$M_A = 3300 \times 21 \times 10.5 + \frac{5950 \times 21}{2} \times 14 - 2000 \times 21 \times 10.5 = 1161300$ lb.ft.
$= 13935600$ in.ft.

$M_B = 6400 \times 10 \times 5 + \frac{2850 \times 10}{2} \times 6.7 - 2000 \times 10 \times 5 = 315475$ lb.ft.
$= 3785700$ in. lb.

A.

$d = 16.5'$ $h = 17.5'$

$R = \frac{13935,600}{12 \times 198 \times 198} = 29.6$

$f_s = 16000$ $f_c = 300$

$p = .002$ $A_s = 12 \times 198 \times .002 = 4.75$

Use $1\frac{1}{4}$"□ bars, 4" cts. 4.69

B.

$d = 8.5$

$R = \frac{3785700}{12 \times 102 \times 102} = 30.3$

$p = .002$

$A_s = 12 \times 102 \times .002 = 2.45$

Use $1\frac{1}{4}$"□ bars, 8" cts = 2.34

EARTH PRESSURE $P = w(H^2 - h^2)C_p$ $x = \frac{(H-h)^2}{6(H+h)}$

$C_p = .19$ $H = 30$ $h = 8$

$P = 100 \times 836 \times .19 = 15900^{\#}$ $x = \frac{22^2}{6 \times 38} = 2.12'$

WING WALLS See diagram for abutment No. 1.

Vert. slab A-B. $M = .064\,ph^3 = .064 \times 14.7 \times \overline{28}^3 \times 12 = 248000$

$f_s = 16000$ $d = 16"$ $f_c = 540$

$R = \frac{248000}{12 \times 16 \times 16} = 80.5$ $pj = .00504$ $p = .565\%$

$A_s = .0056 \times 12 \times 16 = 1.08$ Use $\frac{3}{4}$"□ bars 6" cts.

$P = \frac{1}{2} \times 14.7 \times 28 \times 28 = 5775$ $R = 2290$

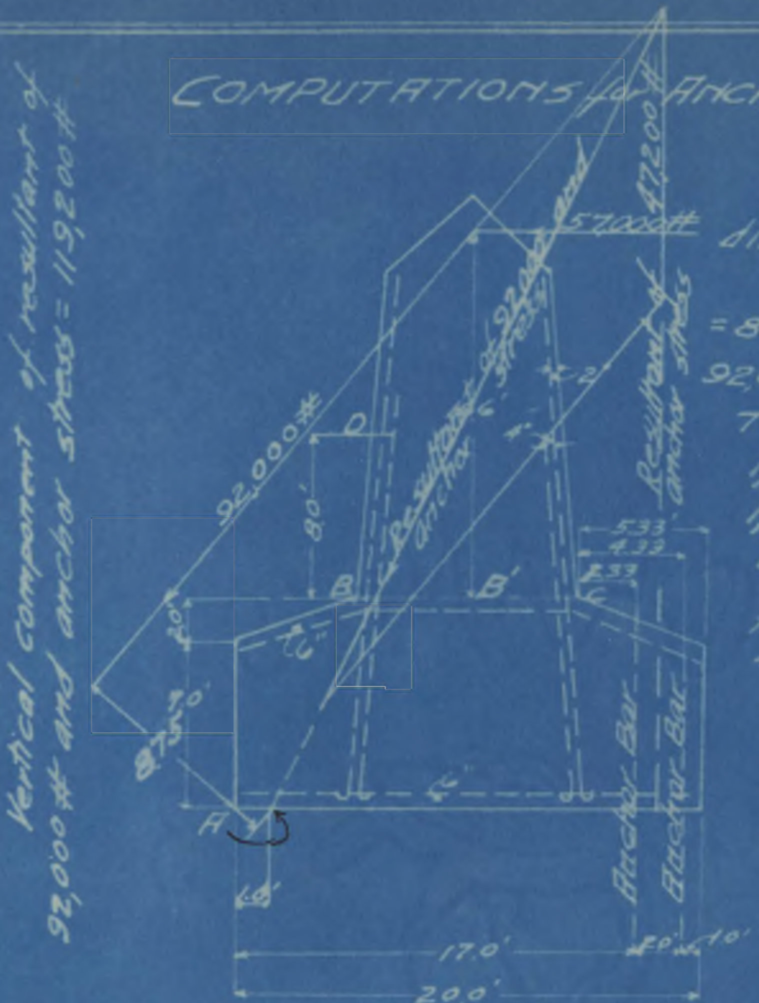
$M = \frac{1}{8} \times 2290 \times 10 \times 10 \times 12 = 344000$ $R = \frac{344000}{18 \times 16 \times 16} = 74.5$

$f_s = 16000$ $p = .0052$ $A_s = .0052 \times 16 \times 18 = 1.5$ sq. in.

Use 3 - $\frac{3}{4}$"□ bars in B-D

" " " " " " strut.

COMPUTATIONS for ANCHOR PIER



T = pull on anchor bar distant unity from A

Bond stress on anchors = 80# □"

$92{,}000 \times 8.75 = T \times 16^2 + T \times 18^2$

T = 1,388 lbs

1,388 × 16 = 22,200#

1,388 × 18 = 25,000#

Use $1\frac{1}{4}$" twisted sq. bars for anchors 12" c'ts

$4 \times 1\frac{1}{4} \times 80 \times L = 25{,}000$

$L = 62\frac{1}{2}$" set anchors 6' deep in rock foundation

Toe pressure $= \frac{2}{3} \times \frac{119{,}200}{1}$

79,500# per sq. ft.

Effective base = 3.0'

COMPUTATIONS for REINFORCEMENT

n = 15, fs = 20,000 # □" (Use Plate # 3 in Turneaure and Maurer)

Mom about C

$12 \times (25{,}000 \times 4.33 + 22{,}200 \times 2.33) = 1{,}918{,}800$ lb ins = M

d = 7.5' = effective depth = 90"

$R = \frac{M}{bd^2} = \frac{1{,}918{,}800}{12 \times 8{,}100} = 19.7$ P = 0.00090

$A_s = 0.00090 \times 12 \times 7.5 \times 12 = 0.97$ □"

Use 1" sq. twisted bars 12" c't's in top of footing

Mom about B

$12 \times 119{,}200 \times 4.33 = 6{,}190{,}000$ lb ins.

d = 98"

$R = \frac{6{,}190{,}000}{12 \times 98 \times 98} = 53.0$ p = 0.0028

$A_s = 12 \times 98 \times 0.0028 = 3.29$ □"

Use $1\frac{1}{4}$" sq. twisted bars 6" c't's

COMPUTATIONS for ANCHOR PIER

Vertical Steel

Moms about B $\qquad f_s = 20{,}000$ #□"

$M = 57{,}000 \times 16.6 \times 12 = 11{,}347{,}000$ lb. ins.

$d = 104'' =$ effective depth

$R = \frac{M}{bd^2} = \frac{11{,}347{,}000}{12 \times 104 \times 104} = 87.5 \qquad p = 0.49\ \% = 0.0049$

$A_s = 12 \times 104 \times 0.0049 = 6.12$ □"

Use 2 layers of $1\frac{1}{4}''$ bars 6" cts. (sq. twisted bars)

Mom. about D

$M = 57{,}000 \times 8.3 \times 12 = 5{,}673{,}500$ lb. ins. $\qquad d = 88''$

$R = \frac{5{,}673{,}500}{12 \times 88 \times 88} = 61 \qquad p = 0.0031$

$A_s = 12 \times 88 \times 0.0031 = 3.38$ □"

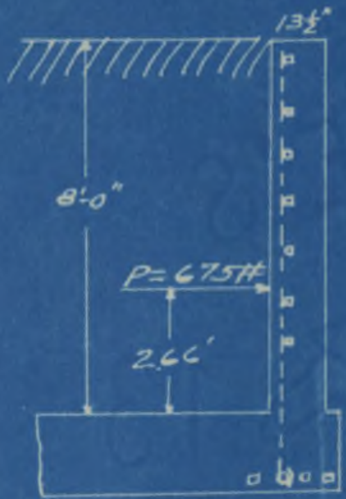
Use $1\frac{1}{4}''$ sq. twisted bars 6" cts.

For diagram of forces acting on the anchor pier see graphics sheet.

Anchor pier is designed to take unbalanced dead load thrust.

Values of R and P are taken from plate #3 in Turneaure and Maurer's "Principles of Reinforced Concrete Construction"

ARCH SPANDREL WALL



Use an equivalent fluid pressure of 21.1 #°'

$P = \frac{1}{2} \times 21.1 \times \overline{8}^2 = 675\#$

$M = 675 \times 2.66 \times 12 = 21{,}600$ lb. ins.

$f_s = 16{,}000\#$°" $d = 11.5$" eff. depth

$R = \dfrac{21{,}600}{12 \times 11.5 \times 11.5} = 13.6$

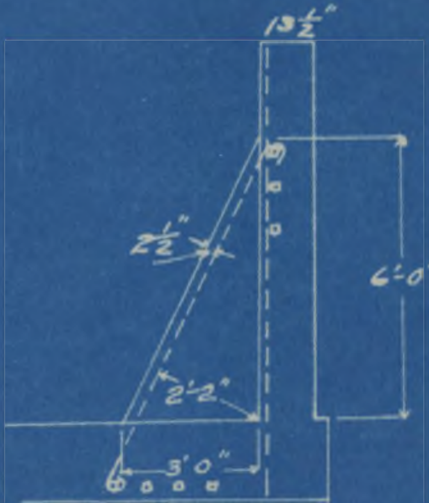
$p = 0.09\ \% = 0.0009$

$A_s = 0.0009 \times 12 \times 11.5 = 0.124$ °"

Use ½" sq. twisted bars vertically 12" on centers.

Use ½" sq. twisted bars horizontally on 12" centers for temperature stresses.

Counterforts



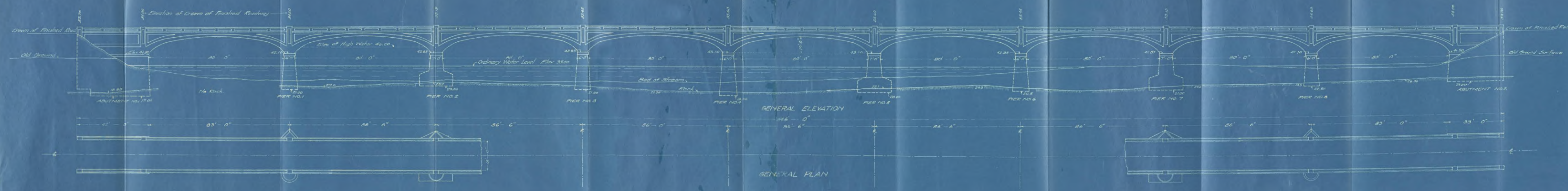
Assume counterfort to support pressure from 7' of earth on wall 10' long.

$M = 42.2\ h^3 = 14{,}500$ lb ins.

$A_s \times 16{,}000 \times 26 = 14{,}500 \times 10$

$A_s = \dfrac{14500 \times 10}{16{,}000 \times 26} = 0.35$ °"

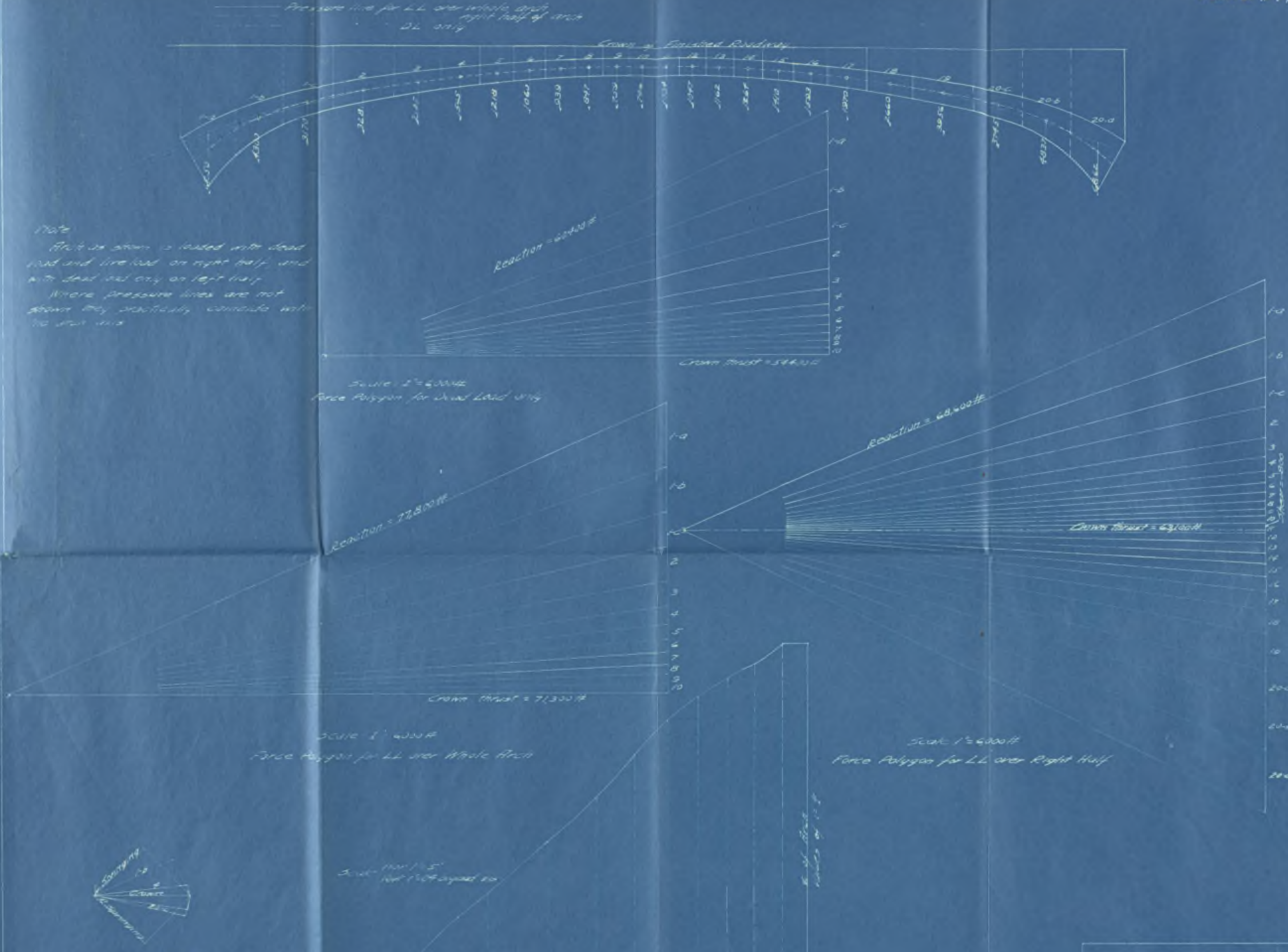
Use 3-½" sq. twisted bars in back of counterfort and make counterfort 12" thick.



PLANS
FOR
REINFORCED CONCRETE ARCH BRIDGE
OVER
ROCK RIVER
AT
COLONA ILLINOIS

Designed by J. H. Anderson and F. T. Sheets
Drawn by F. T. Sheets
June 1914.

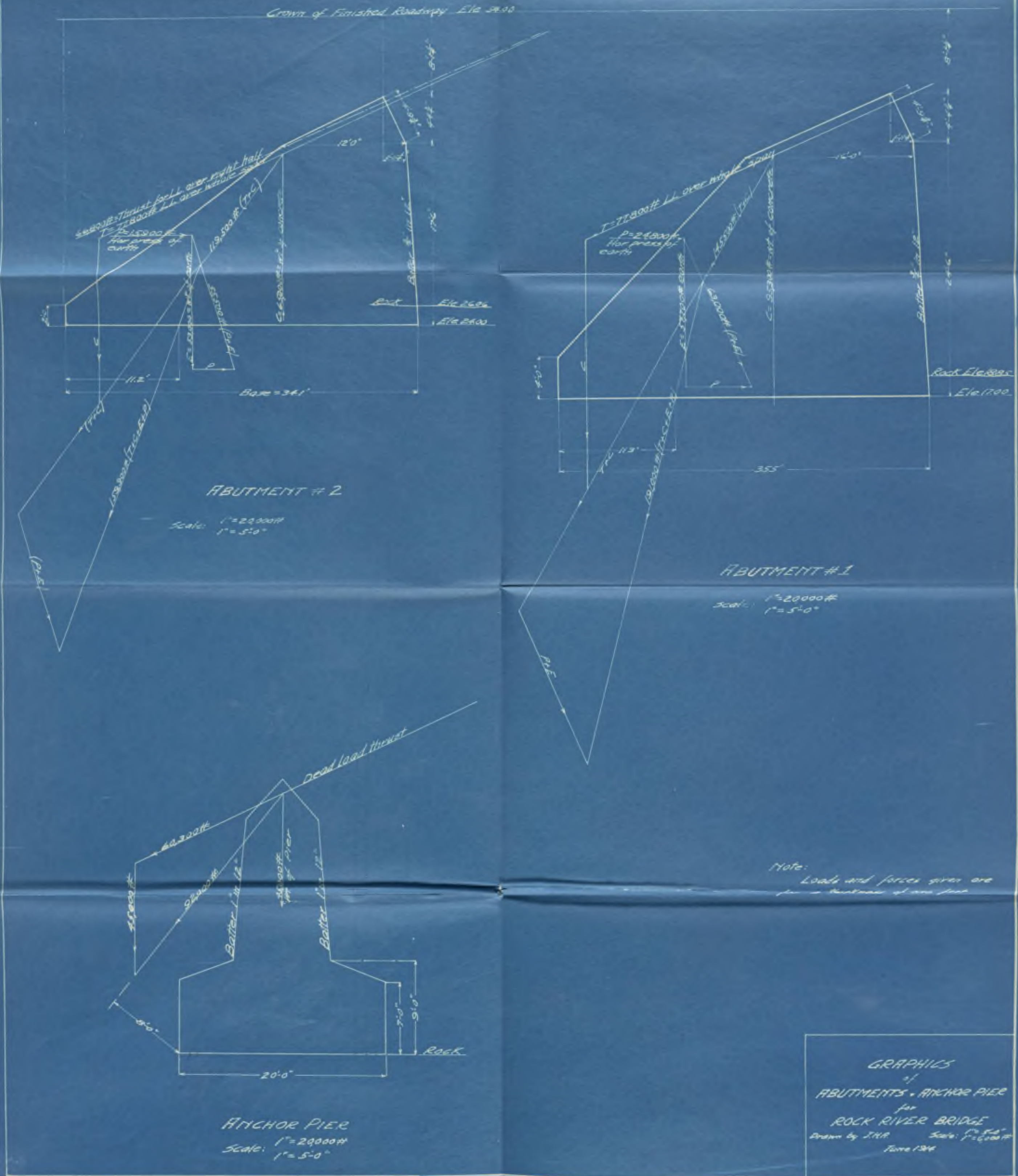
PLATE # 17



GRAPHICS
of
ARCH RING
for
ROCK RIVER BRIDGE

Drawn by JHA — Scale 1" = 5'-0", 1" = 6000#

June 1914


Crown of Finished Roadway Ele 2400
12'0"
Rock
Ele 2606
Ele 2400
11.2'
Base = 34.1'
ABUTMENT # 2
Scale: 1" = 20,000#
1" = 5'-0"
T = 77800# L.L. over whole span
P = 24800#
Hor press of earth
Rock Ele 1885
Ele 1700
35.5'
ABUTMENT # 1
Scale: 1" = 20000#
1" = 5'-0"
Dead load thrust
60,300#
Batter
Rock
20'0"
ANCHOR PIER
Scale: 1" = 20,000#
1" = 5'-0"
Note:
Loads and forces given are
GRAPHICS
of
ABUTMENTS & ANCHOR PIER
for
ROCK RIVER BRIDGE
Drawn by I.H.R.
June 1914

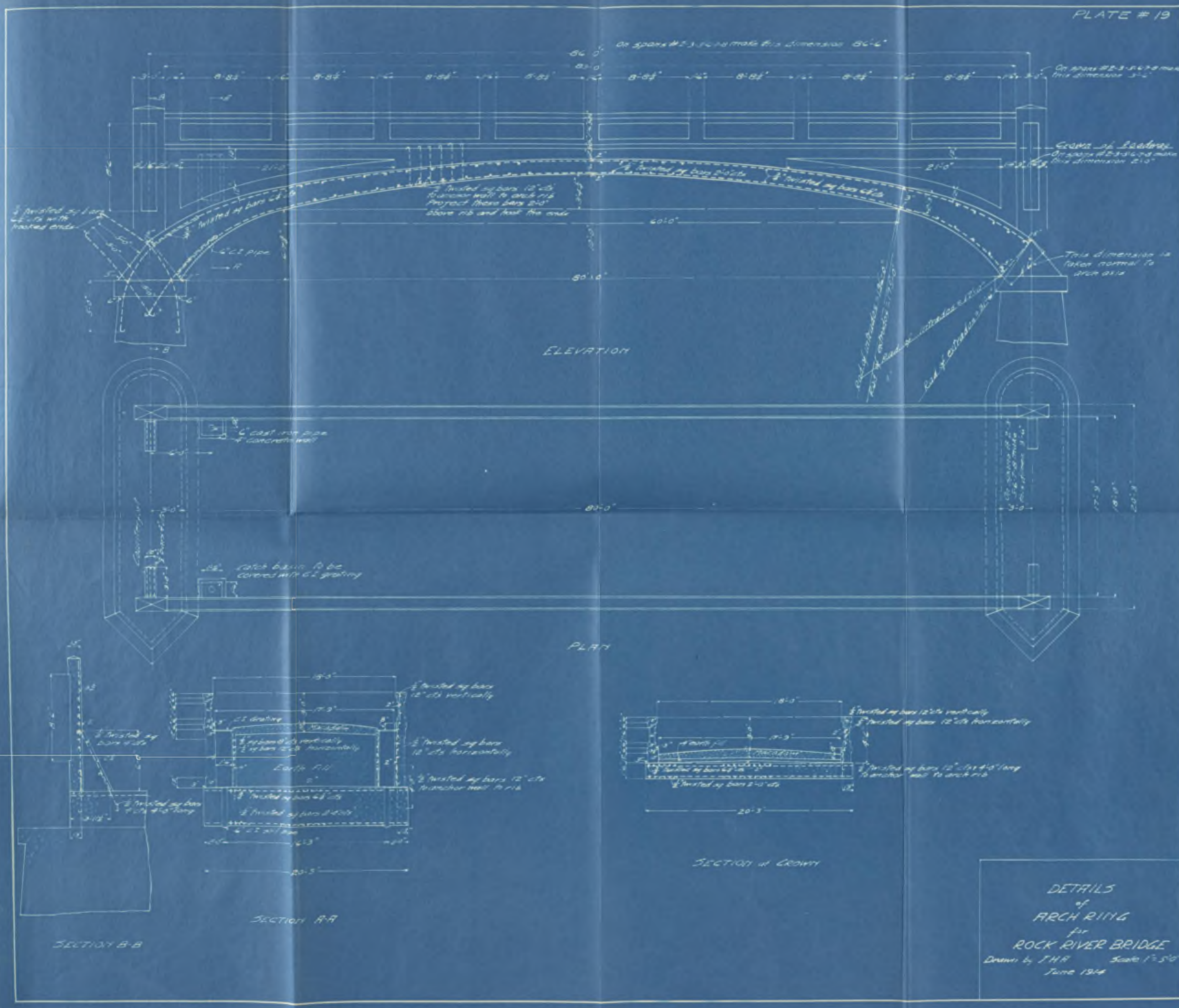
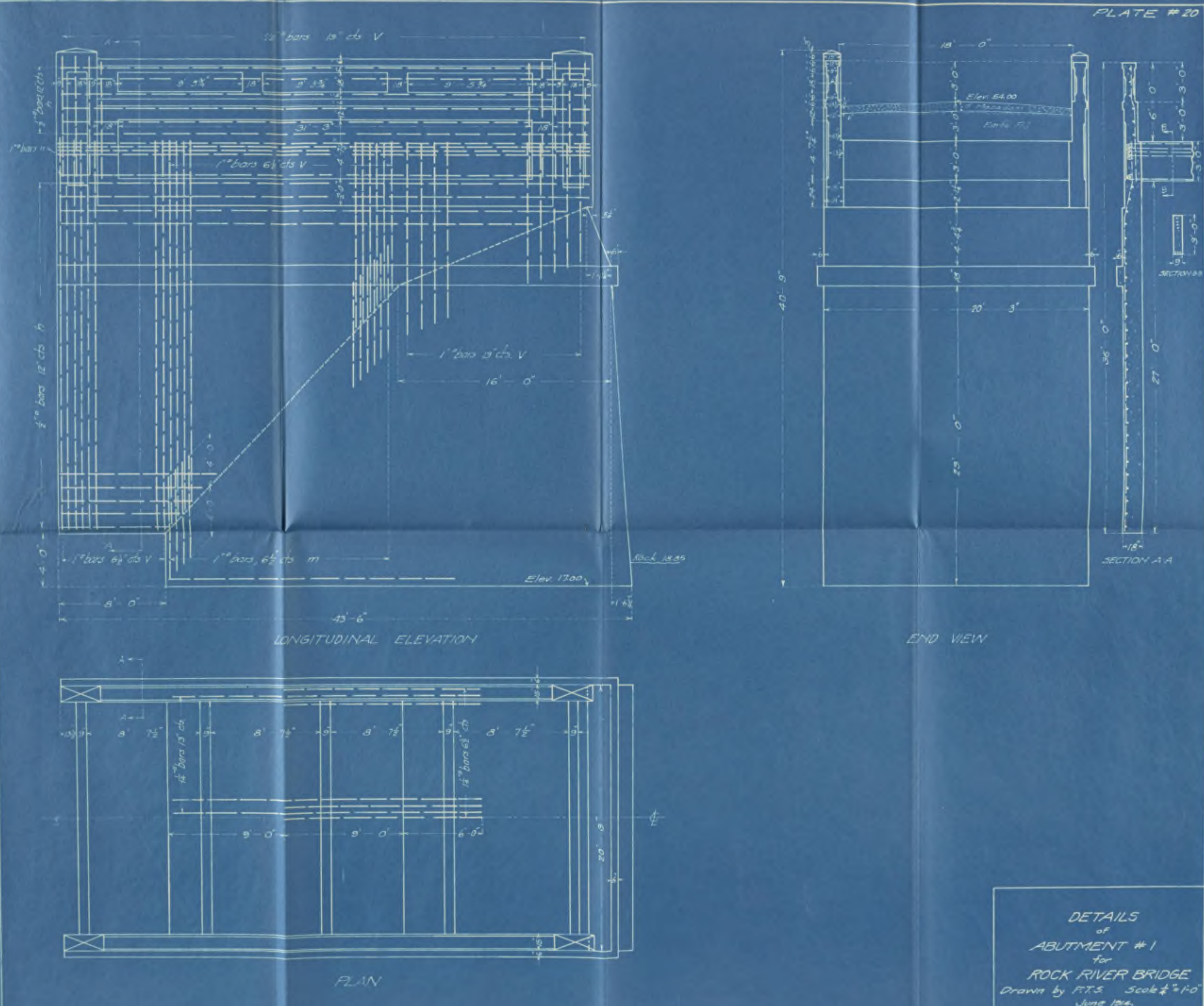


PLATE # 19
On spans #2-3-5-6-8 make this dimension 86'-6"
Crown of Roadway
This dimension is taken normal to arch axis
¾" twisted sq bars 12" c/s to anchor wall to arch rib
Project these bars 2'-0" above rib and hook the ends
4" C.I. pipe
60'-0"
80'-0"
ELEVATION
6" cast iron pipe
4" Concrete wall
Catch basin to be covered with C.I. grating
PLAN
Earth Fill
Macadam
SECTION B-B
SECTION A-A
SECTION at CROWN
20'-3"
DETAILS
of
ARCH RING
for
ROCK RIVER BRIDGE
Drawn by JHR
Scale 1"=5'0"
June 1914



DETAILS
of
ABUTMENT # 1
for
ROCK RIVER BRIDGE
Drawn by F.T.S. Scale ¼"=1'-0"
June 1914.

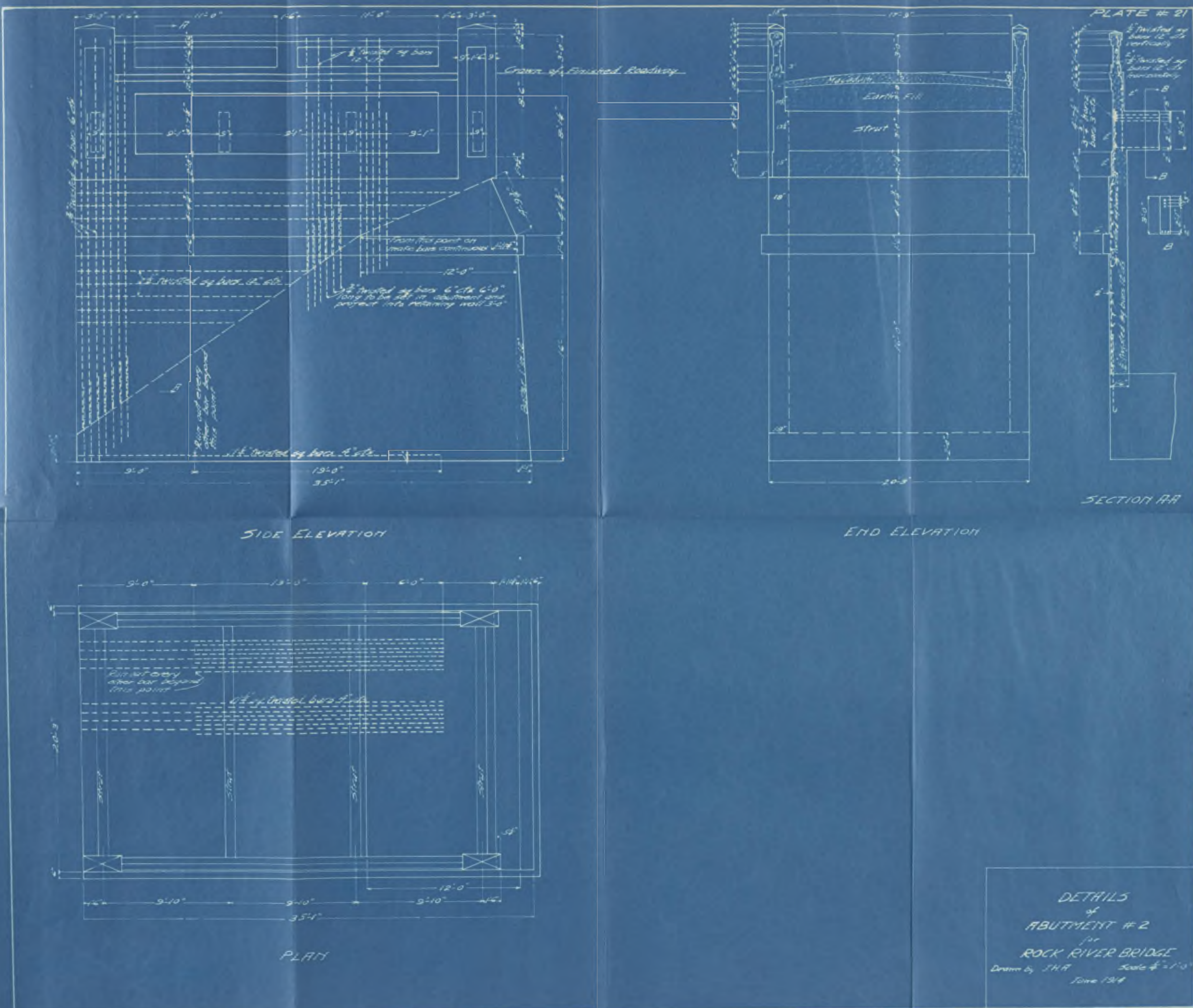


PLATE # 21
Crown of Finished Roadway
Macadam
Earth Fill
Strut
From this point on make bars continuous
12'-0"
9'-0"
19'-0"
35'-1"
20'-3"
SIDE ELEVATION
END ELEVATION
SECTION A-A
PLAN
Strut
12'-0"
9'-10"
35'-4"
DETAILS
of
ABUTMENT # 2
for
ROCK RIVER BRIDGE
Drawn by J.H.A.
Scale 1/4" = 1'-0"
June 1914

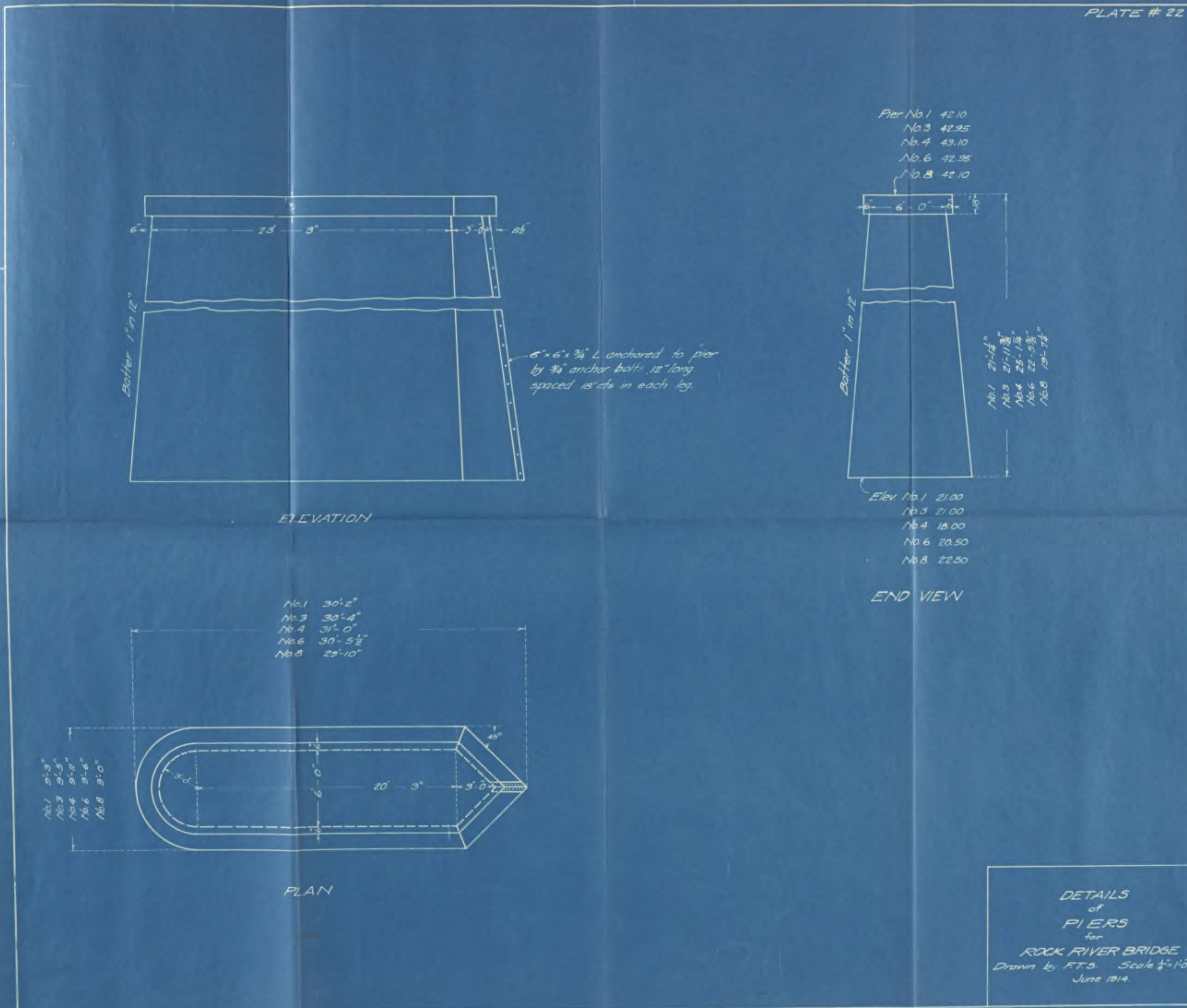


PLATE # 22
Pier No. 1 42.10
No. 3 42.95
No. 4 43.10
No. 6 42.95
No. 8 42.10
6"
23' 3"
3'-0"
8½"
Batter 1" in 12"
6" × 6" × ¾" L anchored to pier
by ¾" anchor bolts 12" long
spaced 18" cts in each leg.
ELEVATION
6' 0"
18"
Batter 1" in 12"
No. 1 21-1¼"
No. 3 21-11⅜"
No. 4 25-1¼"
No. 6 22-5⅜"
No. 8 19-7¼"
Elev. No. 1 21.00
No. 3 21.00
No. 4 18.00
No. 6 20.50
No. 8 22.50
END VIEW
No. 1 30'-2"
No. 3 30'-4"
No. 4 31'-0"
No. 6 30'-5½"
No. 8 29'-10"
No. 1 9'-3"
No. 3 9'-5"
No. 4 9'-11"
No. 6 9'-6"
No. 8 9'-0"
45°
3'-0"
6'-0"
20' 3"
3'-0"
PLAN
DETAILS
of
PIERS
for
ROCK RIVER BRIDGE
Drawn by F.T.S. Scale ¼" = 1'-0"
June 1914.

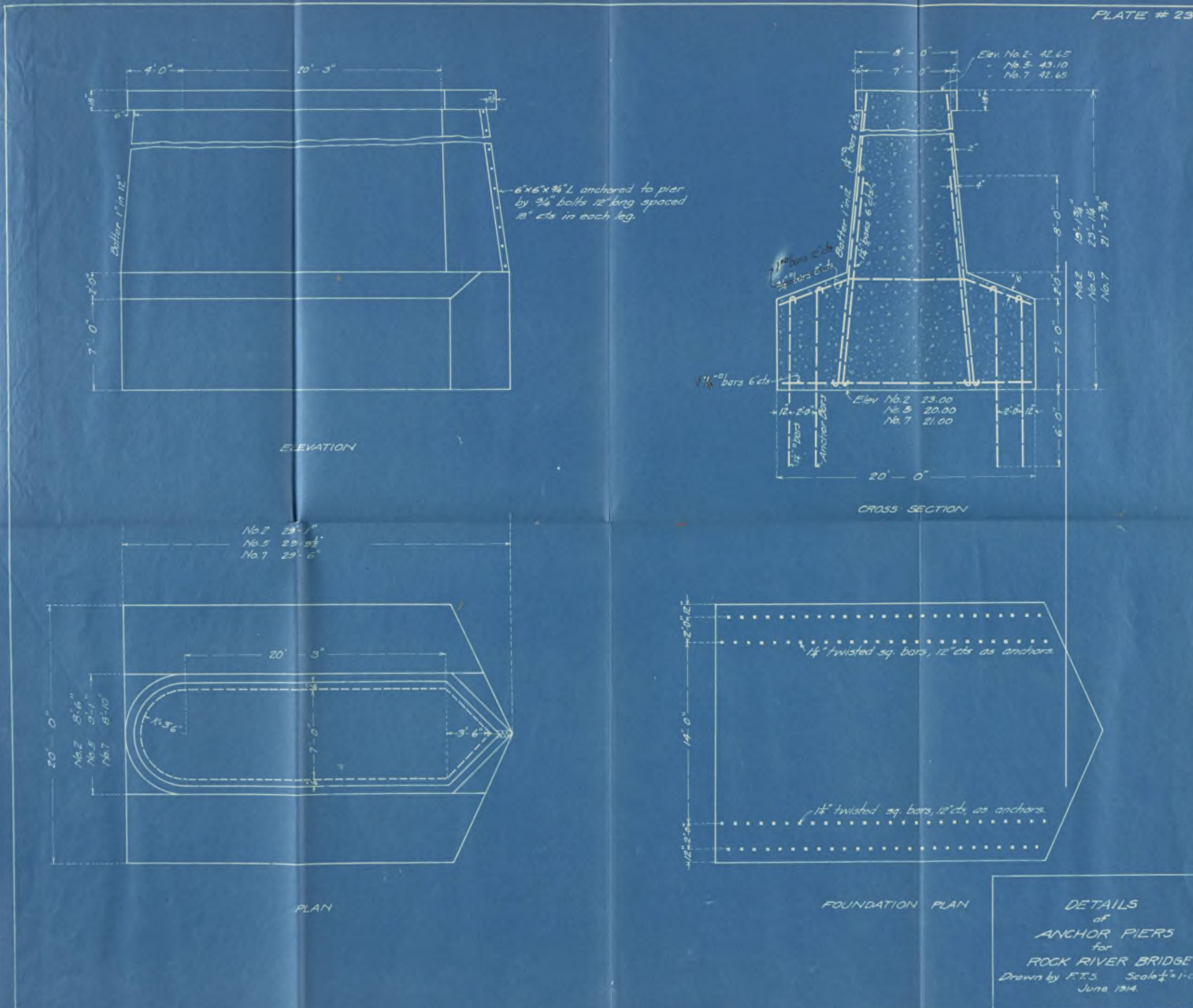


PLATE # 23
6"x6"x3/4" L anchored to pier by 3/4" bolts 12" long spaced 18" cts in each leg.
Batter 1" in 12"
ELEVATION
Elev. No.2 - 42.65
No.5 - 43.10
No.7 42.65
Elev No.2 23.00
No.5 20.00
No.7 21.00
Anchor Bars
20' - 0"
CROSS SECTION
No.2 29'-1"
No.5 29'-9 1/2"
No.7 29'-6"
20' - 3"
20' - 0"
PLAN
1 1/4" twisted sq. bars, 12" cts as anchors.
14' 0"
FOUNDATION PLAN
DETAILS
of
ANCHOR PIERS
for
ROCK RIVER BRIDGE
Drawn by F.T.S. Scale 1/4" = 1'-0"
June 1914.